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CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

OCTOBER, 1953. Vol. XLVIII, No. 10

FORTY-EIGHTH YEAR OF PUBLICATION

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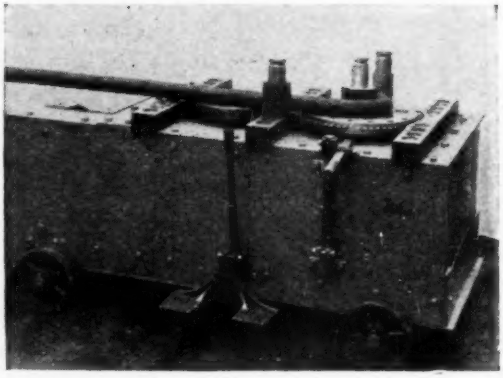

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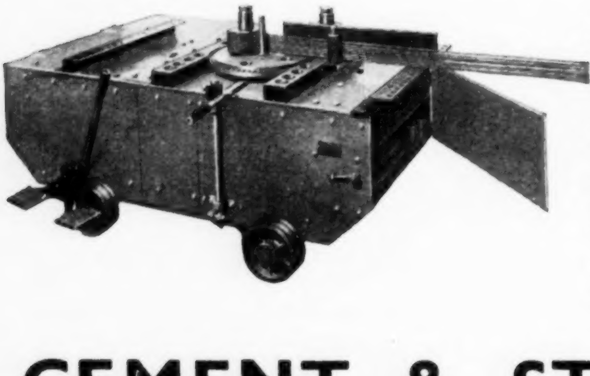

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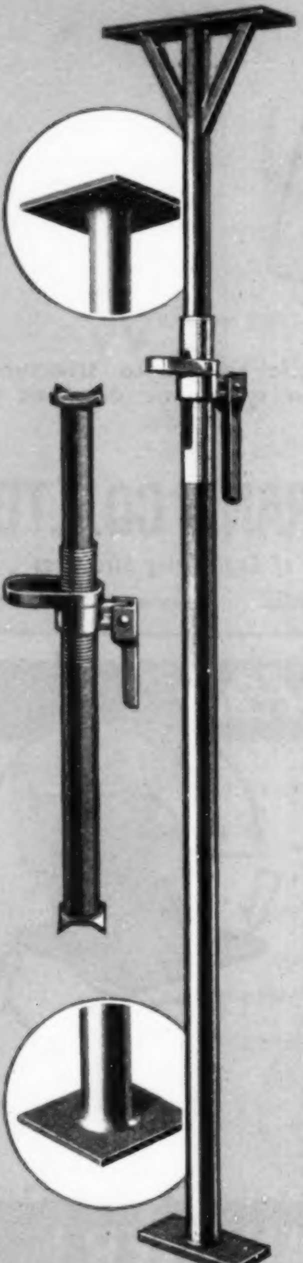

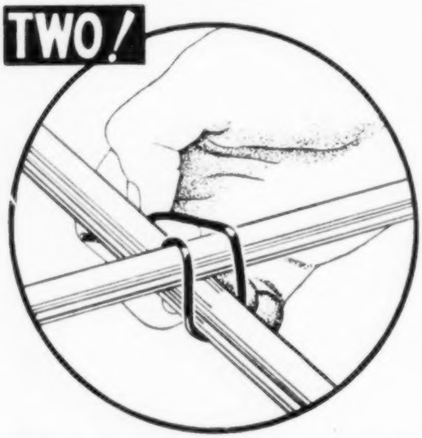

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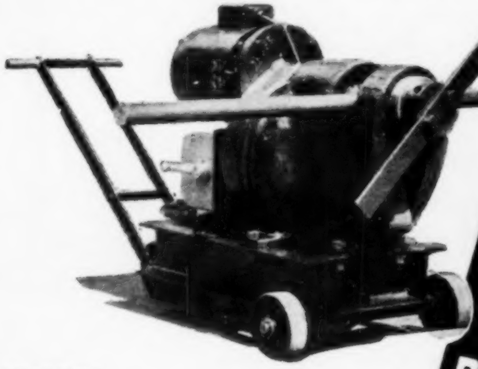

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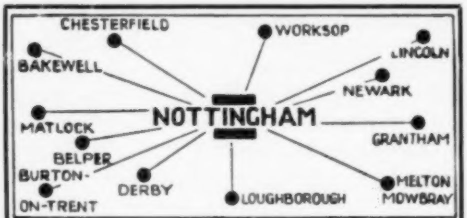

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Volume XLVIII, No. 10. LONDON, OCTOBER, 1953.

EDITORIAL NOTES

Concrete of Uniform Strength.

AMONG the consequences of the steadily increasing cost of building labour and building materials are the efforts made to use less of both. The contractor can influence the cost of concrete by careful planning and the use of better methods in mixing and placing the material, but as often as not he is disappointed to find such savings quickly disappear as the result of increases in the cost of materials and of wages soon after the machinery has been installed. It is therefore natural that more attention should be given to reducing the quantity of concrete used. The obvious method is to make concrete of higher strength so that the same or a greater load can be carried by a smaller or the same quantity. In recent years improved methods of making and placing concrete have resulted in considerably higher strengths being more often obtained. Even with the most efficient processes now in use, however, the variation of strength is so great that it is necessary to assume that, with a reasonable margin of safety, the stress that is allowed is an absurdly small proportion of the highest strength achieved or even of the average strength. Although still higher strengths must be aimed at, it seems that at present it is even more important to strive to make concrete of more uniform strength, so that the lowest strengths will be nearer the average.

This problem is the subject of a paper * recently presented to the Institution of Civil Engineers by the Director of Research of the Danish National Institute of Building Research. In this paper a study is made of the cost of reducing the variability of the compressive strength of concrete, based upon a statistical analysis of the variability of the compressive strengths of cubes from single batches and from different batches. An investigation is made into the cost of the extra care necessary in the selection of materials and the making of concrete so that the variability in strength is reduced and consequently, for the same working stress, concrete of lower average crushing strength may be used. The author estimates that it might cost as much as 3*s.* 4*d.* per cubic yard to reduce the deviation from the average strength from 20 per cent. to 15 per cent. It is also estimated that if the deviation from the average strength were reduced by 10 per cent., and if this more uniform concrete permitted concrete with an average strength of 2680 lb. per square inch instead of 4270 lb. per square inch to be used, there would be a saving in cement of 9*s.* 6*d.* per cubic yard. It is, however, the author's view that the cost of improved methods that would result in a

* "Quality Control of Concrete—Its Rational Basis and Economic Aspects." By N. M. Plum. Journal of the Institution of Civil Engineers, May, 1953.

reduction of more than 5 per cent. in the deviation from the average strength would be more than could be saved by the use of leaner mixtures.

The author states that concrete placed in a Danish airfield varied in compressive strength at seven days from 3000 lb. to over 6000 lb. per square inch during a period of eight months. Also it was found that of three samples taken at the same time, at intervals during the period, the strength varied by nearly 2000 lb. per square inch; in one case the strength of one of the samples was about 4500 lb. per square inch and of the other two more than 6000 lb. per square inch, while in another case two samples had a strength of a little more than 4000 lb. per square inch and the other of more than 6000 lb. per square inch. Seldom was there a difference of less than 700 lb. or 800 lb. per square inch between the strongest and the weakest of three samples made at the same time. This experience is not unusual.

The paper also gives the results of some tests made in Denmark on the efficiency of concrete mixers. These tests indicated that there was a coefficient of variability within each batch of 4 per cent. to 5 per cent. in the case of positive mixers with a vertical axis and one type of non-tilting mixer, and of 8 per cent. or 9 per cent. in the case of what are described as ordinary non-tilting mixers. These variations may not, of course, be due only to the mechanical action of the machines; for example the variations may have been less if the period of mixing had been longer, or the differences in the test results may have been due to differences in the making and testing of the samples. In considering differences in the results of tests on samples taken from different batches still further possibilities may affect the results, such as differences in proportioning, grading, and water content.

Lack of uniformity of strength is common to all building materials, although in the case of concrete the differences are greater than in many other synthetic materials due to the variations in all the materials of which it is made and to the difficulty (or perhaps the impossibility) of ensuring that all the site processes of measuring, mixing, placing, consolidating, and curing are carried out in the same manner throughout the work. It is not difficult to ensure that no concrete will have less than a required minimum strength. To do so, however, at present necessitates a much higher average strength, and if this is achieved some of the concrete will still have perhaps five or six times the required minimum strength. This is very wasteful, for it is the possible lowest strength that has to be taken into account in deciding the working stress that is permissible. Laboratory work may help, but the problems are essentially practical ones of making cement of uniform strength, using aggregates of uniform strength and shape, grading and proportioning materials and water accurately, and uniformity in transporting, placing, compacting, and curing. A means of testing that would give a more reliable indication of the strength of the concrete in a structure than do the preliminary and works cube or cylinder tests now in use is also necessary. These test-pieces are seldom representative of the concrete in a structure, or related to the shape and size of the member, or the distribution of stresses in the structure.

To ensure high-quality concrete of uniform strength it is neither necessary nor desirable that the engineer or contractor should use higher mathematics, as in this paper. Statistical analyses and the theory of probability may be useful in laboratory work but are not likely to appeal to engineers and contractors.

Design of Cantilever Slab Bridges with Stiffened Kerbs.

By ANDREW GALLIA, A.M.I.E. Australia.

A METHOD of designing portal frame slab bridges with stiffened kerbs was given in this journal for December, 1952. In the following a similar method is applied to bridges with a central span and two end cantilevers and stiffened by kerbs on two edges. Reference is made to *Figs.* 4, 5, 6, 9, 10 and 11 of the previous article.

The main features of such a construction are (see *Fig.* 1) : (a) Reduction of the bending moments at midspan of the central span due to the bending moments at the supports of the cantilevers. This allows a reduction to be made in the depth of the central slab or an increase in the length of the span. It is essential that the free ends of the cantilevers should not be supported so that they may transfer the bending moments to the central span ; (b) Reduction of the bending moments in the slabs in the central span and the cantilevers by the use of kerbs as structural members ; (c) The use of narrow piers, attached wing walls, simple bearings, and the saving in excavation make the structure economical. *Fig.* 2 shows three possible conditions of support. $H = 0$ represents a slab with limited width and symmetrical cantilevers ; $H = \infty$ represents a similar structure with rigid edges, and the condition of $0 < H < \infty$ falls between these extremes. *Fig.* 3 gives the position of wheel loads for determining bending moments due to live loads.

Example.

Effective span of central slab, $a = 23$ ft. 6 in. Effective spans of cantilever slabs, $a_1 = 6$ ft. 6 in. Effective width of slabs (the distance from centre to centre of kerbs), $b = 23$ ft. 6 in. Allowable f_s, 18,000 lb. per square inch ; allowable f_c, 900 lb. per square inch ; $m = 12$.

Assume a trial thickness of slab of 14 in. at the crown and $11\frac{1}{4}$ in. at the kerb. Assume the width of the kerb b_1 to be 18 in. and the height of the kerb h_1 to be $25\frac{1}{4}$ in. Then

$$H = \frac{b_1(1-\mu^2)}{a}\left(\frac{h_1}{h}\right)^3 = \frac{18 \times 0{\cdot}98}{23{\cdot}5 \times 12} \times \left(\frac{25{\cdot}25}{12{\cdot}62}\right)^3 = 0{\cdot}50$$

where H is the stiffness factor of the kerb and slab, μ is Poisson's ratio (assumed to be 0·15), and h is the average thickness of slab ($12\frac{5}{8}$ in.).

Dead Loads: Weight of slab and wearing surface (p) 180 lb. per square foot. Weight of kerb and handrail (q) 540 lb. per linear foot.

Live Load : Single 20-tons wheel load (including impact).

Maximum Positive Bending Moment on Slab in Central Span.

(a) Free bending moments on the central span under full load.

Uniformly-distributed load (weight of slab and wearing surface). From the bottom of *Fig.* 4 of the first article, for a ratio of sides of slab $\frac{b}{a} = 1$, $K = \infty$ (representing the freely-supported condition) and $H = 0{\cdot}5$,

$$M = C \times p \times a^2 \times \frac{1}{24} = 1{\cdot}84 \times 180 \times 23{\cdot}5^2 \times \frac{1}{24} = 7600 \text{ ft.-lb.}$$

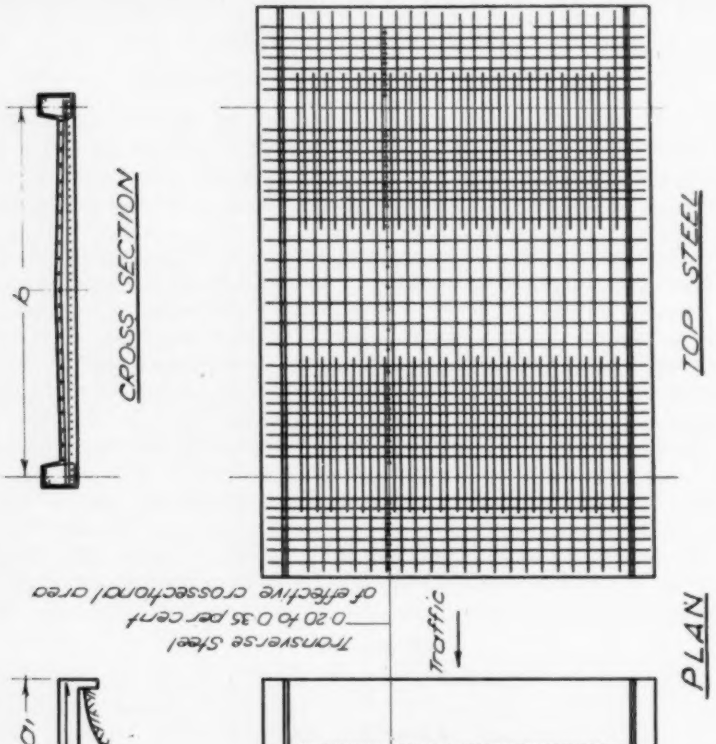

CROSS SECTION
b
TOP STEEL
Transverse Steel
0.20 to 0.35 per cent
of effective crossectional area
Traffic
PLAN
LONGITUDINAL SECTION
a₁
a
W.L
BOTTOM STEEL
Width of Roadway

Fig. 1.

Line load (weight of kerb and fence). From the bottom of *Fig.* 5 of the first article, for $\frac{b}{a} = 1$, $K = \infty$ and $H = 0{\cdot}5$,

$$M = C \times q \times a = 0{\cdot}09 \times 540 \times 23{\cdot}5 = 1150 \text{ ft.-lb.}$$

Live load. One 20-tons wheel load (including impact). From the bottom of *Fig.* 6 of the first article, for $\frac{b}{a} = 1$, $K = \infty$ and $H = 0{\cdot}5$,

$$M = C \times P = 0{\cdot}37 \times 20 \times 2240 = 16{,}600 \text{ ft.-lb.}$$

The total free bending moment is therefore 25,350 ft.-lb.

(*b*) Minimum Bending Moments on the Cantilevers.

Uniformly distributed load (weight of cantilever slab and wearing surface).

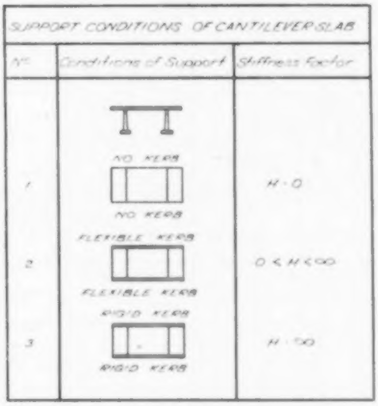


Fig. 2.

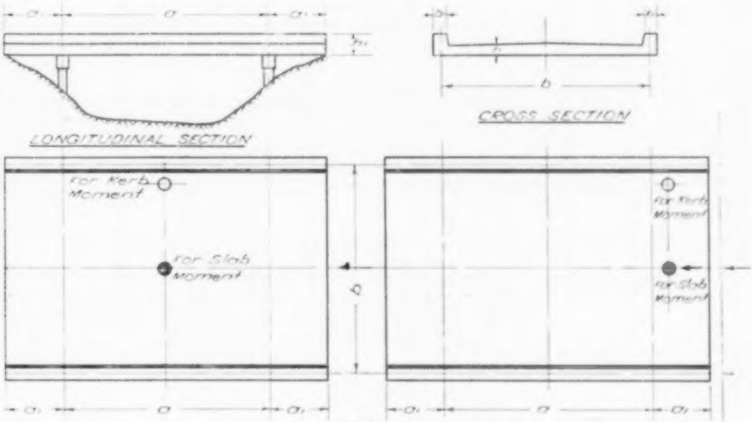


Fig. 3.

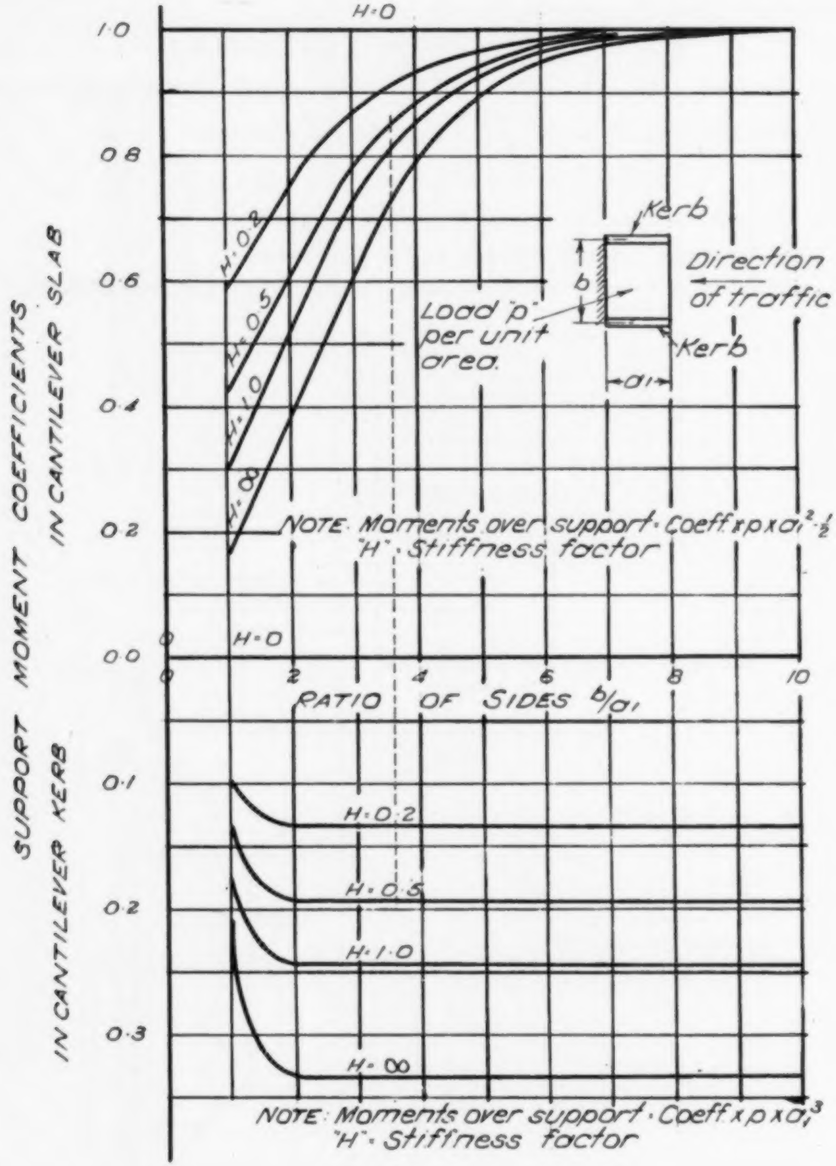

SUPPORT MOMENT COEFFICIENTS IN CANTILEVER SLAB
IN CANTILEVER KERB
H=0
H=0.2
H=0.5
H=1.0
H=∞
Kerb
Direction of traffic
b
Load "p" per unit area.
a1
NOTE: Moments over support = Coeff. x p x a1² ½
"H" = Stiffness factor
RATIO OF SIDES b/a1
NOTE: Moments over support = Coeff. x p x a1³
"H" = Stiffness factor

Fig. 4.

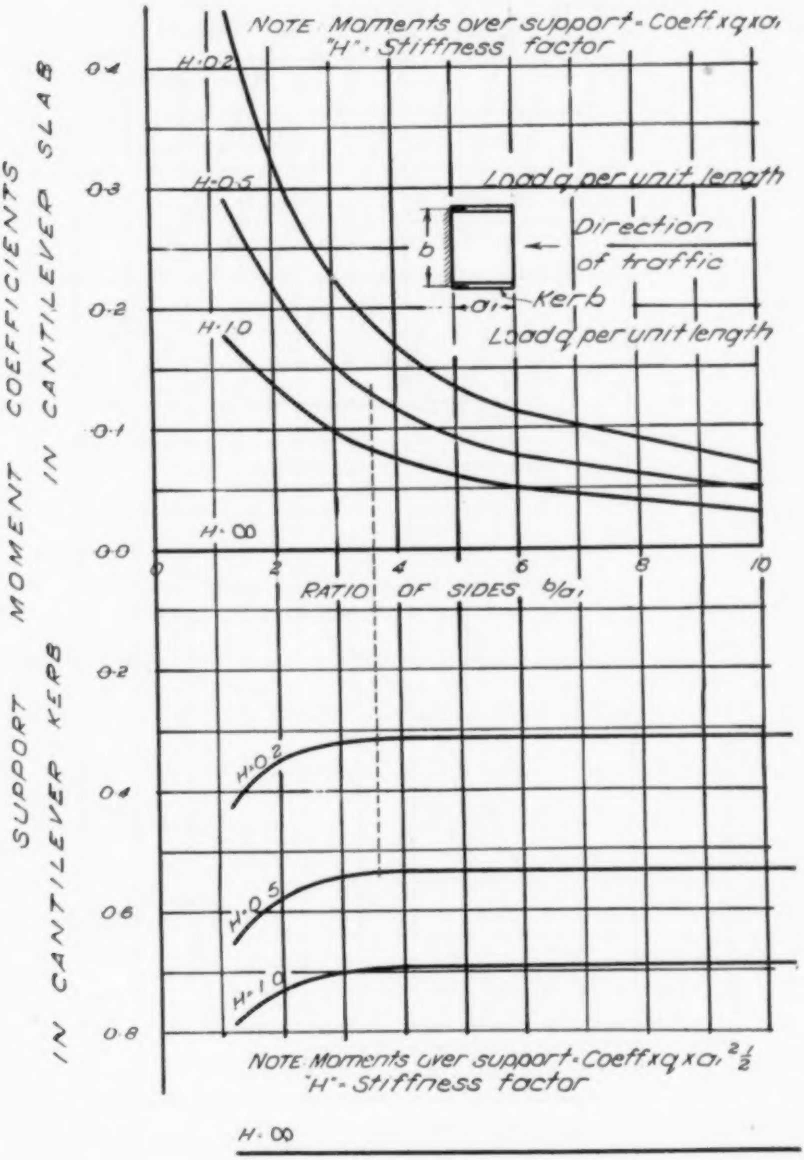

NOTE Moments over support = Coeff x q x a₁
"H" = Stiffness factor
SUPPORT MOMENT COEFFICIENTS IN CANTILEVER SLAB
H·0.2
H·0.5
H·1.0
H·∞
0.4
0.3
0.2
0.1
0.0
Load q per unit length
Direction of traffic
b
a₁
Kerb
Load q per unit length
0
2
4
6
8
10
RATIO OF SIDES b/a₁
IN CANTILEVER KERB
0.2
0.4
0.6
0.8
H·0.2
H·0.5
H·1.0
NOTE Moments over support = Coeff x q x a₁ ²/2
"H" = Stiffness factor
H·∞

Fig. 5.

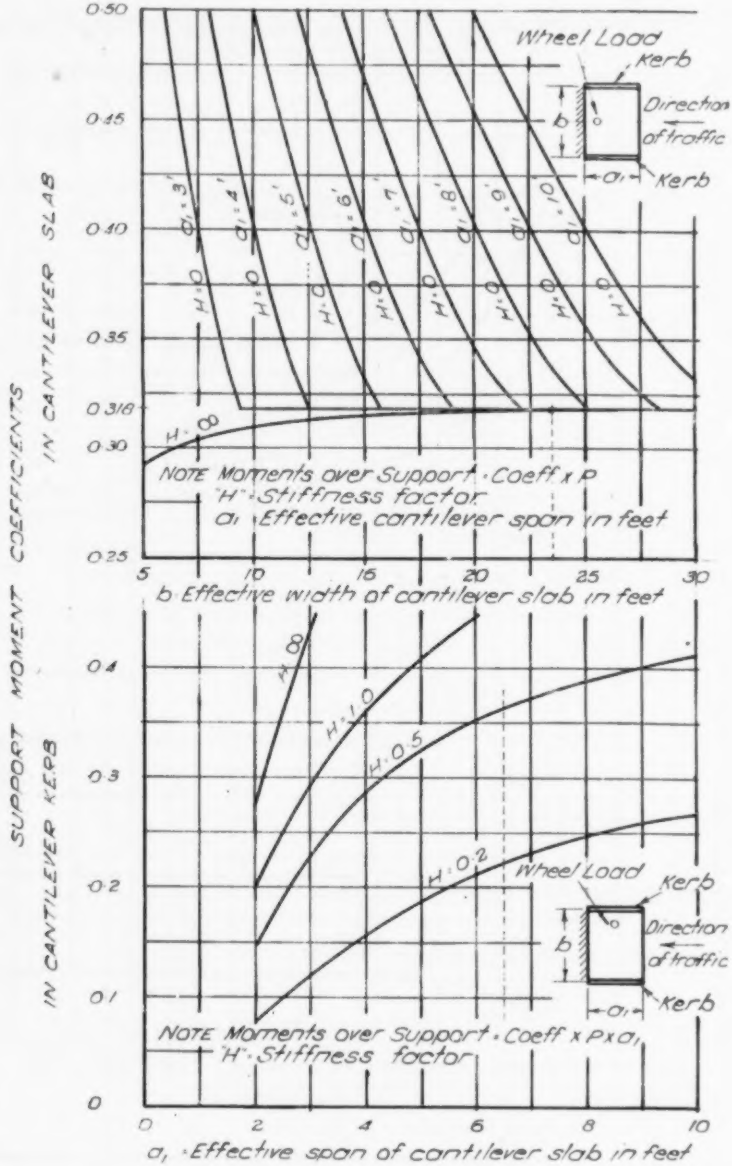

SUPPORT MOMENT COEFFICIENTS IN CANTILEVER SLAB
IN CANTILEVER KERB
Wheel Load
Kerb
Direction of traffic
NOTE Moments over Support = Coeff x P
"H" = Stiffness factor
a₁ = Effective cantilever span in feet
b Effective width of cantilever slab in feet
NOTE Moments over Support = Coeff x P x a₁
"H" = Stiffness factor
a₁ = Effective span of cantilever slab in feet

Fig. 6.

From *Fig.* 4 (top), for $\frac{b}{a_1} = \frac{23\cdot5}{6\cdot5} = 3\cdot6$ and $H = 0\cdot5$.

$M = -C \times p \times a_1^2 \times \frac{1}{2} = -0\cdot85 \times 180 \times 6\cdot5^2 \times \frac{1}{2} = -3240$ ft.-lb.

Line load (weight of cantilever kerb and handrail). From *Fig.* 5 (top), for $\frac{b}{a_1} = 3\cdot6$ and $H = 0\cdot5$.

$M = -C \times q \times a_1 = -0\cdot13 \times 540 \times 6\cdot5 = -455$ ft.-lb.

Curtain Wall. $M = -150 \times 6\cdot25 = -940$ ft.-lb.

Therefore the maximum positive bending moment on the slab of the central span = 25,350 − 4635 = 20,715 ft.-lb.

MAXIMUM POSITIVE BENDING MOMENT ON KERB IN CENTRAL SPAN.

(a) Free bending moments on the central span under full load.

Uniformly-distributed load on the slab (weight of slab and wearing surface). From the bottom of *Fig.* 9 of the first article, for $\frac{b}{a} = 1$, $K = \infty$ and $H = 0\cdot5$,

$M = C \times p \times a^3 = 0\cdot028 \times 180 \times 23\cdot5^3 = 65{,}200$ ft.-lb.

Line load (weight of kerb and handrail). From the bottom of *Fig.* 10 of the first article, for $\frac{b}{a} = 1$, $K = \infty$ and $H = 0\cdot5$,

$M = C \times q \times a^3 \times \frac{1}{24} = 1\cdot73 \times 540 \times 23\cdot5^2 \times \frac{1}{24} = 21{,}600$ ft.-lb.

Live load.—One 20-tons wheel load (including impact). From the bottom of *Fig.* 11 of the first article, $a = 23$ ft. 6 in., $K = \infty$, $H = 0\cdot5$,

$M = C \times P \times a \times \frac{1}{8} = 0\cdot89 \times 20 \times 2240 \times 23\cdot5 \times \frac{1}{8} = 117{,}000$ ft.-lb.

The total free bending moment is therefore 203,800 ft.-lb.

(*b*) Minimum bending moments on the cantilevers.

Uniformly-distributed load (weight of cantilever slab and wearing surface). From *Fig.* 4 (bottom), for $\frac{b}{a_1} = 3\cdot6$ and $H = 0\cdot5$,

$M = -C \times p \times a_1^3 = -0\cdot19 \times 180 \times 6\cdot5^3 = -9400$ ft.-lb.

Line load (weight of cantilever kerb and fence). From *Fig.* 5 (bottom), for $\frac{b}{a_1} = 3\cdot6$, and $H = 0\cdot5$,

$M = -C \times q \times a_1^2 \times \frac{1}{2} = -0\cdot54 \times 540 \times 6\cdot5^2 \times \frac{1}{2} = -6150$ ft.-lb.

Curtain Wall. $M = -(150 \times 1\cdot5 \times 6\cdot25) - (525 \times 8) = -5610$ ft.-lb.

Therefore the maximum positive bending moment on the kerb in the central span = 203,800 − 21,160 = 182,640 ft.-lb.

MAXIMUM BENDING MOMENT ON CANTILEVER SLAB.

Bending moment due to uniformly-distributed load (as before)	= −	3240 ft.-lb.
,, line load (as before)	= −	455 ft.-lb.
,, curtain wall (as before)	= −	940 ft.-lb.
,, live load. One 20-tons wheel load. From *Fig.* 6 (top) for $b = 23\cdot5$, $a_1 = 6\cdot5$, and by interpolation between $H = 0$ and $H = \infty$,		
$M = -C \times P = -0\cdot318 \times 20 \times 2240$	= −	14,250 ft.-lb.
The maximum bending moment on the cantilever slab is	−	18,885 ft.-lb.

To find the value of the coefficient in *Fig.* 6, enter this diagram from the value of b (effective width of cantilever slab) and proceed vertically. If the curve a_1 (effective cantilever span) falls to the left of the vertical, the required coefficient for $H = 0$ will be 0·318 ; on the other hand, if the curve is to the right of the vertical, the intersection with the vertical will give the required coefficient. Thus

in the case of $b = 23{\cdot}5$ and $a_1 = 6{\cdot}5$ the coefficient for $H = 0$ will be 0·318, but for $a_1 = 8$ the coefficient is 0·337 ; for $a_1 = 9$, the coefficient is 0·383 ; for $a_1 = 10$, the coefficient is 0·428. *Fig.* 6 (top) gives also the maximum economical value of a_1 for the given b ; that is for $b = 23{\cdot}5$, $a_1 = 7{\cdot}5$ would be the most economical effective span for the cantilever.

Maximum Bending Moment on Cantilever Kerb.

Bending moment due to uniformly-distributed load (as before)	= −	9400 ft.-lb.
,, line load (as before)	= −	6150 ft.-lb.
,, curtain wall (as before)	= −	5610 ft.-lb.
,, live load (one 20-tons wheel load). From *Fig.* 6 (bottom), for $a_1 = 6{\cdot}5$ and $H = 0{\cdot}5$,		
$M = -C \times a_1 = -0{\cdot}365 \times 20 \times 2240 \times 6{\cdot}5$	= −	106,500 ft.-lb.
Therefore the maximum bending moment on the cantilever kerb	= −	127,660 ft.-lb.

Maximum Negative Bending Moment on Slab in Central Span.

(a) Free bending moments (dead load only),		
due to uniformly-distributed load (as before)	=	7600 ft.-lb.
,, ,, line load (as before)	=	1150 ft.-lb.
		8750 ft.-lb.
(b) Maximum bending moment on slab of central span due to cantilevers (as before)	= −	18,885 ft.-lb.
Maximum negative bending moment on central slab	= −	10,135 ft.-lb.

Maximum Negative Bending Moment on Kerb in Central Span.

(a) Free bending moments (dead load only),		
due to uniformly-distributed load (as before)	=	65,200 ft.-lb.
,, ,, line load (as before)	=	21,600 ft.-lb.
		86,800 ft.-lb.
(b) Maximum bending moment on cantilever kerb (as before)	= −	127,660 ft.-lb.
Maximum negative bending moment on kerb in central span	= −	40,860 ft.-lb.

To resist the stresses due to the negative bending moments on the slab and the kerbs of the central span, top reinforcement is required throughout the spans. This reinforcement is also used in compression when calculating the stresses at midspan for the maximum positive bending moment, and this makes possible a smaller depth of slab.

Bending moments in the direction transverse to the traffic are not discussed in this analysis and it is suggested that transverse bars as indicated in *Fig.* 1 should be used.

A Symposium on Concrete.

A symposium on the proportioning of mixtures and the control of the quality of concrete is being arranged by the Cement & Concrete Association in conjunction with the Reinforced Concrete Association, the Joint Research Committee of the British Cast Concrete Federation and the Cast Stone and Concrete Federation, and the Prestressed Concrete Development Group, and with the co-operation of the Building Research Station and the Road Research Laboratory. The symposium will take place at the Institution of Civil Engineers from April 6 to 8, 1954. Further details may be obtained from the Cement & Concrete Association, 52 Grosvenor Gardens, London, S.W.1.

A Shell Roof at Edinburgh.

THE type of construction described was chosen to give a large floor area free from columns and to reduce maintenance charges in a building at the chemical works at Edinburgh of Messrs. T. & H. Smith, Ltd. The new building (*Fig.* 1) is 190 ft. long, 60 ft. wide, and 38 ft. high from the floor to the underside of the crown of the roof. The roof is of shell construction of elliptical section as shown in *Fig.* 2. The shells span between reinforced concrete frames at 31 ft. centres, with a large edge-beam at the eaves which forms both a gutter and an architectural feature. The frames project above the shell in order to provide an unobstructed ceiling. Details of the frames are given in *Figs.* 3–5. The reinforcement comprises 1-in. bars and $\frac{3}{8}$-in. stirrups. The cover of concrete is generally $1\frac{1}{2}$ in. The design stresses were 1000 lb. per square inch in the concrete and 18,000 lb. per square inch in the reinforcement. Stock lengths of steel

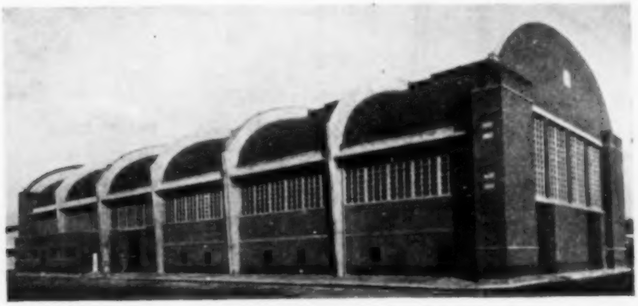


Fig. 1.

Fig. 2.—Typical Cross Section.

were used, resulting in the unsymmetrical arrangement of bars shown. There are two expansion joints through the building, and here twin frames are used ; the joints are made watertight by two thick bituminous-felt water-stops. The top of the frames is covered with a lead cap, and the joint pointed with mastic on the two sides.

The shell is covered with two layers of bituminous roofing felt weighing 7 lb. per square yard and finished with red brickdust. The ceiling has been sprayed with a ½-in. thickness of asbestos to reduce heat losses and to prevent condensation. A colouring pigment was added to the final coat of the spray.

Ventilation is by two extract fans of

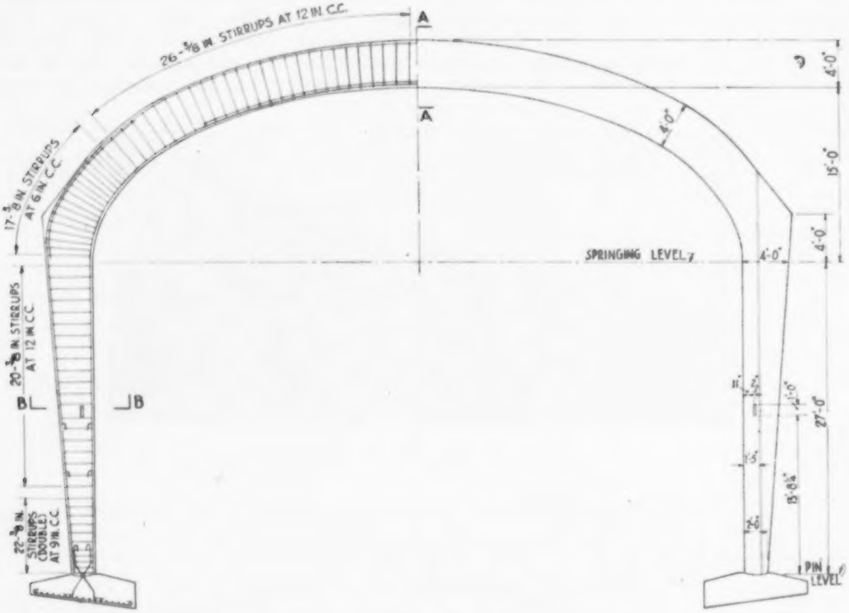


Fig. 3.

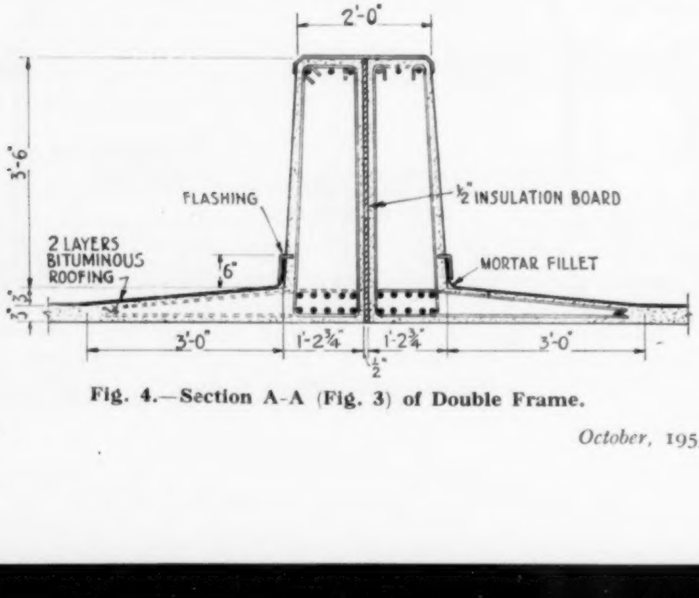
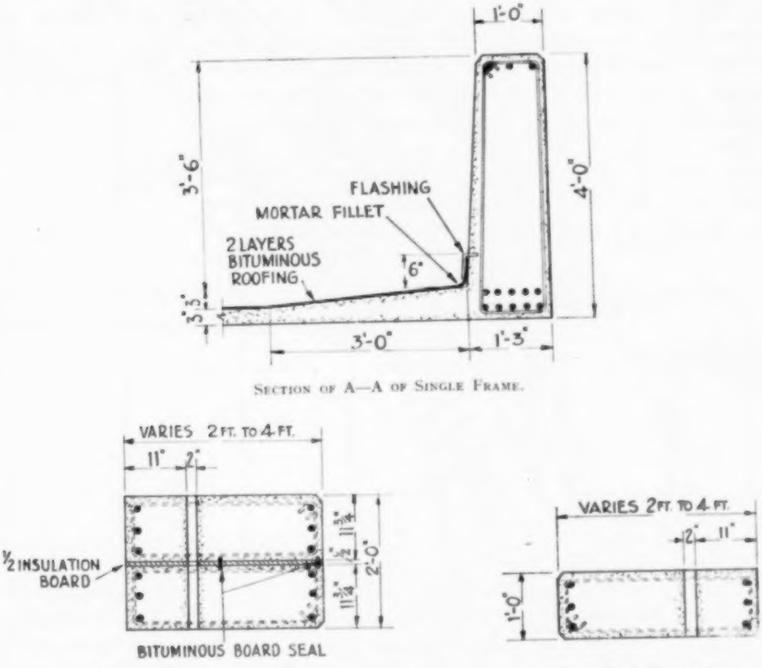


Fig. 4.—Section A-A (Fig. 3) of Double Frame.

2 ft. 6 in. diameter in the roof of each bay. Air is drawn into the building through steam-heated radiators built into the brick walls 2 ft. above floor level. The electric cables for the fan motors and for all lighting points were laid on top of the shell concrete before waterproofing was carried out, and hangers were built into the shell to carry the lighting fittings. Natural lighting is by a central roof-light of glass-and-concrete profile tubes. Enough shuttering was provided for two complete bays of 31 ft., so that the shuttering for one bay could be erected while the concrete in the previous bay was hardening.

The extrados of the roof was shuttered to a height of 12 ft. In this case the shuttering was arranged so that it could be quickly erected for successive lifts of concrete. Ribs were cut from 6-in. by 2-in. timber to the profile of the extrados



Section of A—A of Single Frame.

Section B-B of Double Frame.

Section B-B of Single Frame.

Fig. 5. *(See Fig. 3 for positions of sections.)*

construction and by continuous side windows with concrete frames.

The method of shuttering the soffit of the shell was unusual, and is shown in *Fig.* 6. Tubular scaffold poles were bent to the profile of the shell and fixed at 2 ft. centres to longitudinal transoms at 4 ft. centres, which were carried by posts at 6 ft. centres. Flexible panels of timber shuttering were made by nailing 6-in. by 1⅛-in. dressed boards to steel strip; these could be easily bent to the required shape and were wired to the and fixed at 2 ft. centres. To the underside of these ribs were nailed steel strips 3½ in. wide, and to the sides were nailed 2-in. by 1-in. timber battens (*Fig.* 6), thus forming a groove in which the shutter boards could be slid.

The concrete was in the proportions of 1 : 2 : 4 and was transported to the roof in containers which were filled at the mixer and travelled on a light-gauge track to a hoist which lifted them through the roof-light near the centre of the building. On the roof, the containers were

discharged into two-wheeled hand carts in which the concrete was distributed.

The erection of frames and shells was completed in nine months and the entire building, including annexes, was completed in twelve months. The structure was designed by Messrs. Blyth & Blyth, MM.I.C.E., of Edinburgh. The reinforced concrete work was carried out by Messrs. Holst & Co., Ltd. Messrs. John Wight & Co. (Edinr.), Ltd., were the general contractors.

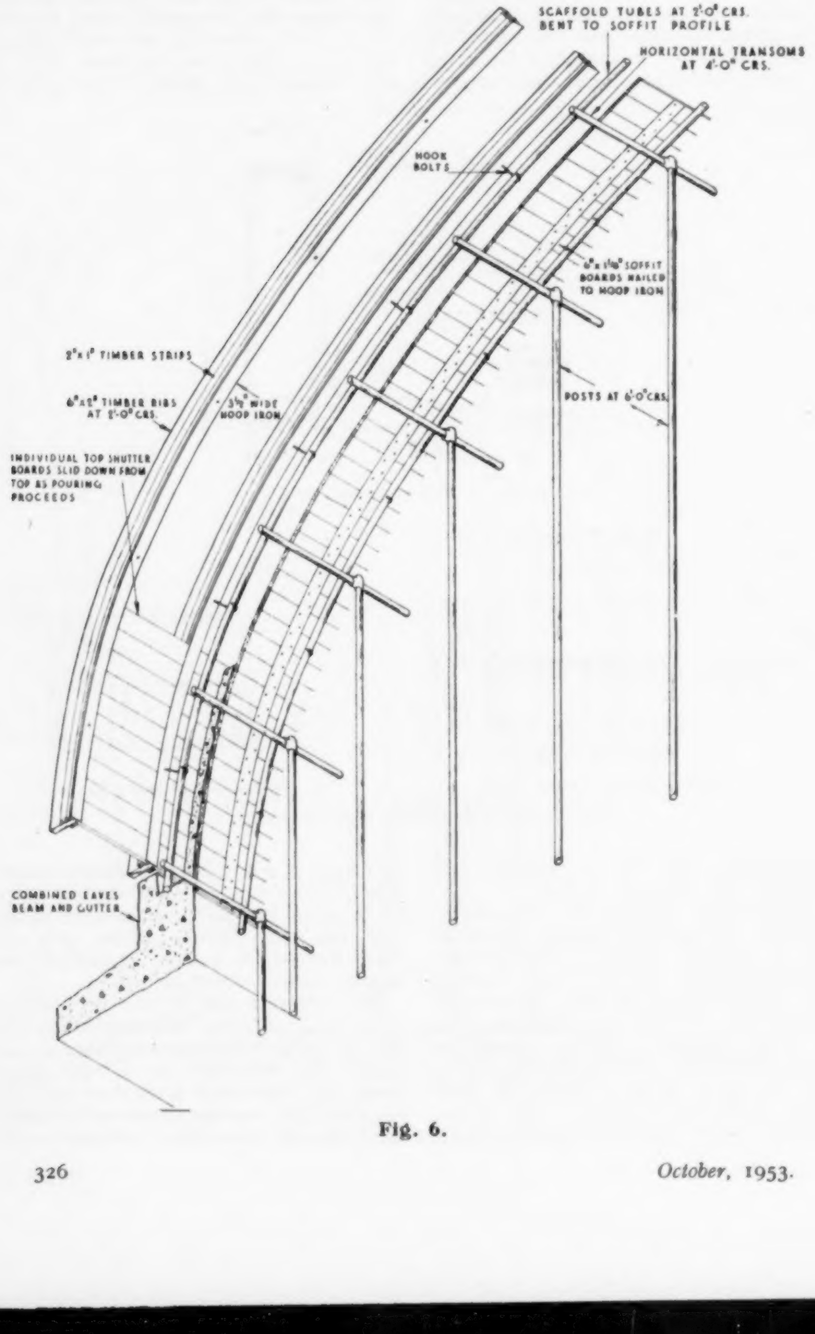


Fig. 6.

The Application of Reinforced Concrete to School Buildings—II.*

By FELIX J. SAMUELY, B.Sc., A.M.I.C.E.

In some cases gymnasia and assembly halls are of two stories and a suspended floor of comparatively long span (40 ft. to 50 ft.) is often required. The conditions imposed for the design of these floors are usually less stringent than those for classrooms. The area of the windows is not usually critical; there is no need to allow for blind-boxes; and, although the floors have to carry a slightly larger uniformly-distributed load, there are no partition walls. Furthermore, particularly in changing-rooms and gymnasia, flat ceilings are not usually required and the cheapest floor construction usually results in the cheapest total construction.

It is, of course, desirable to use the same sizes of precast beams and slabs as for the classrooms in order that the same moulds may be used. This is sometimes possible by using the method adopted at Hatfield Technical College and School, namely by arranging cantilevers on the main frames so that the span of the floor is equal to that of the classrooms (*Fig.* 19). Here the trussed beams used were the same as for the classrooms and, as the grid was also the same, floor troughs of the same size were also used.

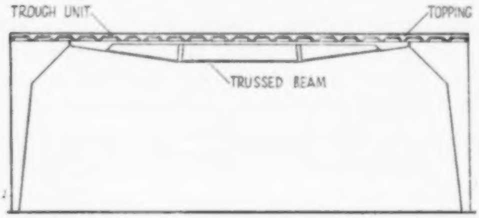


Fig. 19.

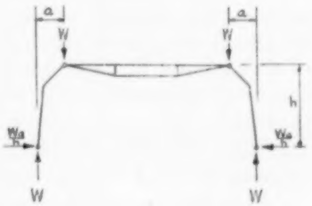


Fig. 20.

In cases such as that shown in *Fig.* 19 the columns and the beam form a four-pinned frame which is unstable in itself. Such frames can, however, be used if the upper slab is prevented from moving horizontally and is held in position at least along two lines by walls or rigid frames, and if the slab itself is capable of transmitting the forces to these walls. In *Fig.* 20 there is no horizontal force to be resisted by the slab if the loads are symmetrical, but the forces may act either way if there are unsymmetrical loads. The resultant force will usually not be very large and can often be resisted by cross walls. This, of course, is possible only with either in-situ or composite construction, as a rigid slab is essential to transmit the forces.

As it is not always possible to use this type of construction the beams have frequently to span the full width of the building. Prestressed concrete planks of the type described in the previous article have been used up to a length of 40 ft., as at Old Palace School, Poplar (*Fig.* 21). An example where prestressed planks, together with troughs, have been kept visible and merely painted on the underside is the changing rooms at Eastcote Lane Secondary Modern School,

* Concluded from September number

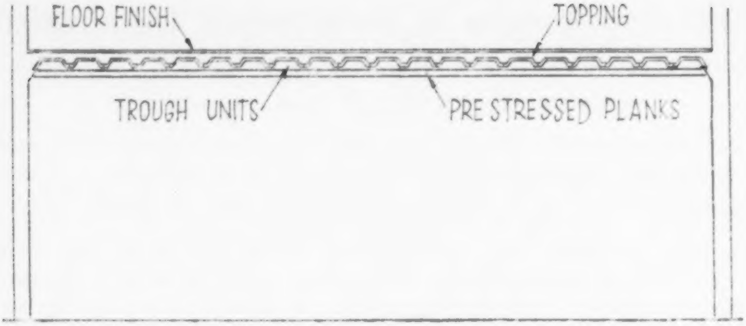


Fig. 21.

Middlesex (*Fig.* 22). In some cases the spans of changing rooms and gymnasia are reduced by load-bearing walls, and in such cases in-situ reinforced concrete beams have been used when the number of beams did not justify their being precast.

Roofs of Gymnasia, Assembly Halls, and Dining Rooms.

The choice between a flat and a sloping roof does not usually rest with the engineer but depends on architectural considerations. For lightly-loaded roofs reinforced concrete is often not an economical material for the slab because of its weight, and in these cases it is preferable to support the roof covering by structural steelwork. If a reinforced concrete roof slab is preferred because of its superior properties compared with some forms of sheeting it is usually more economical to make the beams also of reinforced concrete, even for large spans, for both flat and sloping roofs.

FLAT ROOFS.—There is no doubt that the larger the span the greater becomes the relative economy of prestressed concrete, again because saving in depth

Fig. 22.

means saving in weight. It was mentioned before that for spans up to about 40 ft. prestressed beams with bonded wires were more economical, and prestressed planks have been used in the same way as for classrooms (for example, a roof of 40 ft. span for the assembly hall at a school at Barley Lane, Ilford, *Fig.* 23). In order to use planks of the same cross section as for the classrooms, but without increasing the depth, beams were composed of two standard planks side by side, giving a beam 3 ft. 8 in. wide by 1 ft. deep. Similar construction was adopted

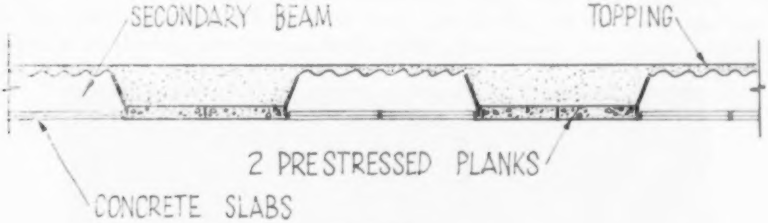


Fig. 23.

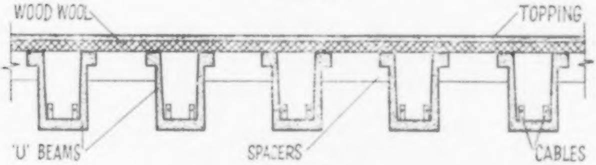


Fig. 24.

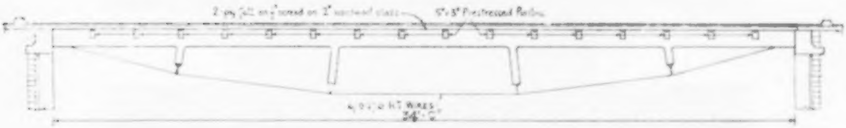


Fig. 25.

for the dining room at Wigan Technical School and the assembly hall at Old Palace School, Poplar.

For spans greater than 40 ft., post-tensioned members have been found more satisfactory than pre-tensioned members. *Fig.* 24 is a cross section through the roof of the gymnasium and assembly hall at a school at Cottingham where the gymnasium spans 40 ft. and the assembly hall 48 ft. The beams of both buildings comprise precast units 8 ft. long which were placed end to end and post-tensioned before erection to form beams of the required length. The precast units are U-shaped so that it is possible to keep the cables in the open and avoid having to form ducts. It will be seen, also, that keeping these cables in the open makes prestressing easier and reduces losses of the prestressing force due to friction. Intermediate diaphragms are arranged so that the cable can be curved slightly, or rather given a series of kinks, and the end diaphragms have been used as abutments for the jacks. The cables are easily grouted after tensioning. The post-tensioned member is then covered with an in-situ topping laid on woodwool slabs, resulting in a comparatively light composite section.

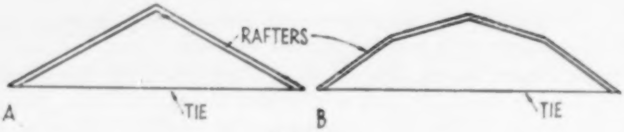


Fig. 26.

Fig. 27.

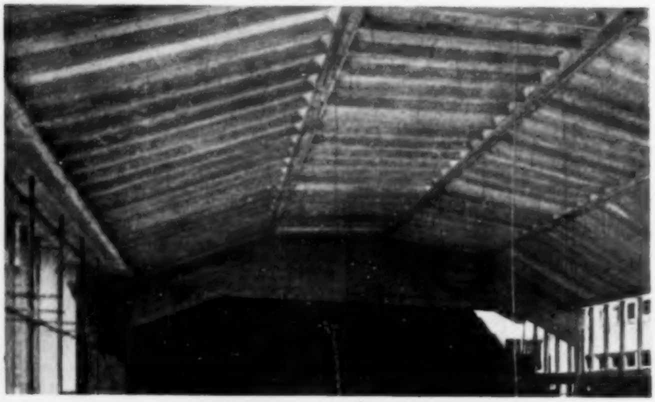


Fig. 28.

Fig. 29.

A special case is a roof spanning 36 ft. for the assembly hall at a school at Dartford, where woodwool slabs on purlins were carried by a special trussed beam (*Fig.* 25). This trussed beam is similar to those discussed in the previous article but consists of five panels (instead of three) with a lower chord of approximately parabolic outline. The lower chords again are cables and were post-tensioned by jacking all four struts.

Sloping Roofs.—Three methods have been found suitable for sloping roofs, namely (a) Rigid frames, (b) Roof trusses, and (c) " Skin " structures.

In-situ rigid frames have recently been almost out of the question due to the shortage of timber, but precast frames have been used, for example, at Hat-

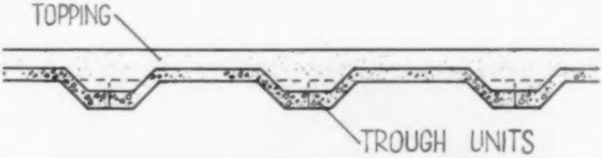


Fig. 30.

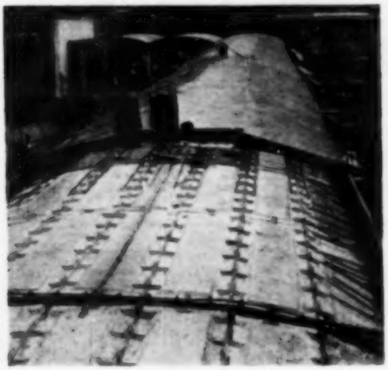


Fig. 31.

field Technical College (see *Fig.* 2, p. 58, " Concrete and Constructional Engineering," February, 1952). The rather small grid of 5 ft. 6 in. was in this case determined by the use of this grid elsewhere, but it is interesting to note that a much smaller grid is economical with precast concrete than with in-situ concrete because the greater repetition reduces the cost of each unit very considerably. Similar frames have been used for the workshop roof at Hatfield College and for primary schools for the Kent County Council.

Roof trusses in reinforced concrete are usually restricted to the shapes shown in *Figs.* 26A and 26B ; because of the ties these are not very popular.

A construction that appears to have great possibilities is the " skin " roof, which is a prismatic thin slab structure. The writer is somewhat sceptical about the use of curved shell roofs for schools because there is rarely sufficient repetition to make the curved shuttering economical. By using a slab as shown in cross

section in *Fig.* 27, which is a composite construction, such shuttering can be avoided. *Fig.* 28 shows an assembly hall at Wigan Technical School. Precast trough units were arranged in four rows, supported temporarily on scaffolding; reinforcement for the slab was placed (*Fig.* 29) and the whole roof then concreted and the struts removed. The use of the troughs allowed a rather thin and light roof having the rigidity of a much thicker roof because of the ribs between the units (*Fig.* 30). It is well known that in such roofs the shearing stresses are of great importance, but as high shearing stresses occur only towards the ends the in-situ concrete could be easily thickened at these positions where the extra weight had the least influence on the bending moments. A similar roof was used for the assembly hall at the Primary School at Kingsmead, Hackney. *Fig.* 31 shows some covered ways at Blackwell Secondary School, Middlesex,

Fig. 32.

where the same principle was used although the troughs in this case were arranged with their long sides parallel to the eaves.

A variation of this method is the use of a latticed construction in precast concrete covered with a lightweight material. A particularly interesting example is the school at Abingdon where two square assembly halls, 40 ft. by 40 ft., were constructed with a pyramidal roof in which a latticed construction in the plane of the roof was arranged (*Fig.* 32). All members are of precast concrete except the diagonal bracings which are of steel and anchored to the concrete.

The foregoing descriptions show that slabs of spans encountered in schools, that is from 24 ft. to 40 ft., are a useful and economic application of prestressed concrete.

The writer does not claim that the designs suggested for floors are completely satisfactory, for such a floor should enable heating services, soil-pipes, and elec-

trical conduits to be laid and renewed at any time within the thickness of the floor. This cannot be done with any of the usual floors of hollow tiles or solid slabs, but it can with the type of floor described consisting of planks and troughs with an accessible space between and closed by a false ceiling. This false ceiling is an expensive item, but an attempt to omit it makes the service space inaccessible. A satisfactory solution of the problem has still to be found.

Prestressed Bars for Reinforcing Concrete.

A NEW method of reinforcing concrete has recently been proposed by Mr. W. M.J. Ryzewski, A.M.I.C.E. The method, for which a patent application has been made, comprises the use of one or more high-tensile wires tensioned and embedded in a concrete member of any convenient cross section. This member is used in place of steel reinforcement subjected to either compressive or tensile stresses. The prestressed members may be in the form of small slabs and used as combined shuttering and reinforcement, but it is thought that the most general application would be in the form of circular bars of any length and of a cross-sectional area of about 1 sq. in.

Such a bar, with a diameter of 1⅛ in., would be prestressed by one to four 0·08-in. high-tensile wires stressed to 200,000 lb. per square inch, the stress in the concrete varying from 1000 lb. to 4000 lb. per square inch according to the number of wires used. The high stresses in the concrete would require the use of high-alumina cement or rapid-hardening Portland cement to make concrete with a strength when stressed of about 12,000 lb. per square inch. The bar would be cast in a mould consisting of the halves of a steel tube of any convenient length. The lengths of bars required for any structure would be cut from the longer bars. It would be necessary to bind the bar at the ends, and on both sides of the position where a cut is to be made, to prevent splitting of the concrete. The binding may be of any suitable material strong enough to withstand the outward pressure due to the expansion of the ends of the wires when the tensioning force is released.

The high-tensile wires are placed so that the resultant force acts along the centre of gravity of the bar and so that there will be no tendency to buckle due to prestressing. Compared with a mild-steel bar stressed to 18,000 lb. per square inch and with a modulus of elasticity of 30,000,000 lb. per square inch, the strain in the concrete bar, stressed to 4000 lb. per square inch and with a modulus of elasticity of about 5,000,000 lb. per square inch, would be 33 per cent. greater, but the cracks would not extend to the wires surrounded by concrete in compression. Designs have been prepared by Mr. Ryzewski to compare normally reinforced members with those reinforced by the proposed method, and it is stated that the latter requires only about one-tenth of the amount of steel and that there is an average saving in the cost of materials of £8 to £10 for each ton of mild steel displaced by the prestressed bars.

Strengthening a Dam by Prestressing.

To increase the capacity of Steenbras reservoir, Cape Town, South Africa, an existing dam is being prestressed to enable its height to be increased without having to increase its thickness. Holes are drilled vertically through the dam and about 30 ft. into the rock below. Into these holes cables are inserted with the bottom ends splayed so that they may be anchored by grouting them into the rock. At the top of the dam the end of a cable is formed into a loop, a concrete block is placed in the loop, and jacks placed between the block and the top of the dam tension the cable. After tensioning is completed the cables are grouted in the hole through the dam. The height of the dam will be increased by building on top of the prestressed structure. The foregoing is abstracted from "Engineering News-Record," August 13, 1953.

Prestressed Precast Beams and Slabs.

Design for Small Footbridges.

In *Fig.* 1 is a design for a bridge composed entirely of members such as are used for the floors and roofs of buildings. The longitudinal members comprise channel-beams with the prestressing wires arranged so that the beams can be used on their sides as shown in the drawing. An extra 2 in. of concrete is cast on one side, and this is used as the upper flange of a beam 1 ft. 6 in. deep by 6 in. wide. In the extra concrete a rebate is formed along one edge to receive slabs, 1 ft. 6 in. wide by 1 in. thick with ribs 2¼ in. deep around the edges, which form the deck. Sockets to receive posts for handrails are also cast in the extra 2 in. of concrete. Similar bridges with widths of up to 7 ft. and spans to about 13 ft. have been designed for light vehicular traffic. In these designs the slabs are placed with the ribs uppermost and concrete is cast on top to form a deck of the thickness required. The beams weigh 54 lb. per foot and contain forty-two No. 12 gauge high-tensile wires. Their greatest moment of resistance is 354,000 in.-lb. when the maximum compressive stress in the concrete is 1680 lb. per square inch. This design is by Milbank Floors, Ltd., whose works at Great Waltham, Essex, is described in the following.

The beams (*Fig.* 3) are of two types, namely, (*a*) a channel-shaped beam 1 ft. 4 in. wide by depths varying from 4 in. to 8 in., and (*b*) hollow beams 6 in. deep by 1 ft. 6 in. wide. Channel-shaped beams 4 in. deep weigh 15 lb. per square foot and are suitable for imposed loads of 30 lb. per square foot on spans up to 15 ft.; beams 8 in. deep weigh 28 lb. per square foot and may be used for heavier loads on spans up to 30 ft., their moment of resistance being 126,000 in.-lb. The hollow beams are made of expanded-clay aggregate and have good insulating properties; they weigh 33 lb. per square foot and are suitable for domestic floors on spans up to 16 ft., and roofs to 18 ft.

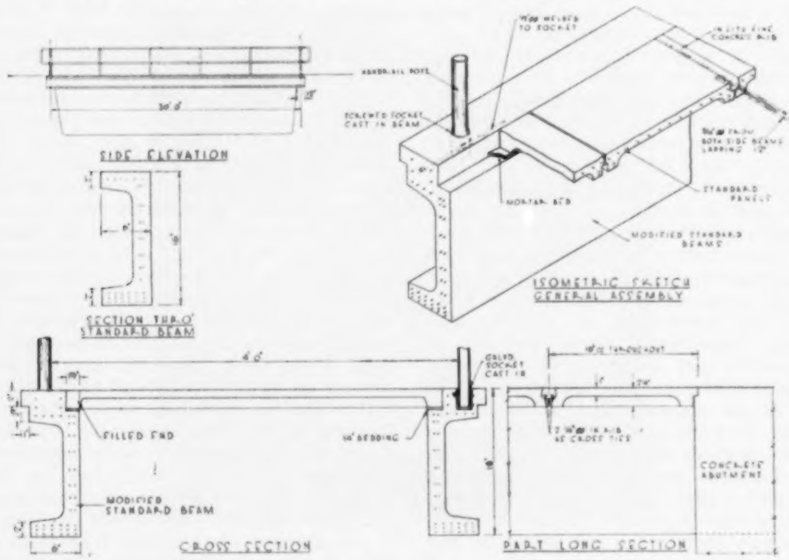


Fig. 1.—Design for Footbridge with Prestressed Beams and Slabs.

Channel-Beams.

The channel-beams are made on production lines 140 ft. and 150 ft. long between the anchorages. On the concrete bearers of the production lines are

Fig. 2.—Stressing Beds for Floor Beams.

fixed hollow concrete mould-bases, which form the channel-shaped soffits of the beams. The mould-bases are of prestressed concrete made in one piece 135 ft. or 145 ft. long; these long prestressed units are found to be subject to much less deterioration than reinforced concrete bases made in 15-ft. lengths. The mould-bases are laid on the bearers in rows of four, five, or six (*Fig.* 2), and concrete mould-sides and dividers in 15-ft. lengths are placed to form the sides of adjacent beams.

Twenty-one production lines are in use for channel-beams, which are made in depths of 4 in., 5 in., 6 in., 7 in., and 8 in. No. 12-gauge high-tensile wire of 120 to 130 tons breaking strain is used and is delivered in coils of 2 ft. diameter. The coils are mounted on spinners, from which as many wires as are needed are led to an assembly frame where they are passed through the anchor-plates and steel plates which form the end of each beam. One anchor-plate is then pulled by a winch to the end of the production line where the wires are wedged to the anchor-plate and cut, and the complete assembly of wires, anchor-plates, and division plates is lifted over the mould-base, the anchor-plates fitting into the fixed frames.

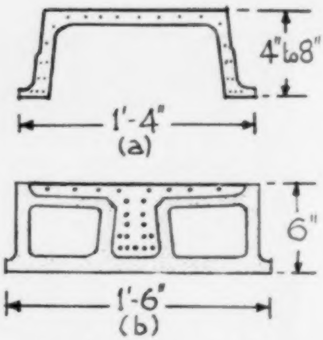


Fig. 3.—Cross Section of Beams.

Initial tension is applied by a hand-operated device (*Fig.* 4) to each wire, or pairs of wires, at one anchor-plate, and the wires are then fixed. The main tension is then applied by an hydraulic jack (*Fig.* 5) pulling on the other anchor-plate, to a fixed extension, where the plate is held by steel distance-pieces. The combination of the required amount of extension and the initial tension, which may be varied at will, allows the required tension to be applied to wires of sometimes varying qualities. When the wires are fully tensioned, the division plates are

Fig. 4.—Applying Initial Tension to Wires.

adjusted to the lengths of beams to be cast, and the mould-sides are placed and fixed, securing the division plates in position.

The concrete is consolidated by vibrators mounted on frames clamped rigidly to the divisions between the beams, along which the frames are moved as concreting progresses. Surface vibration is also applied to the top of the concrete when the moulds are filled.

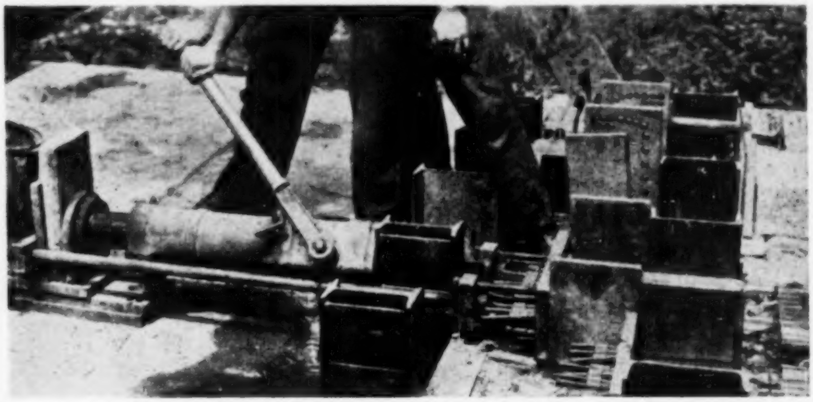


Fig. 5.—Applying Final Tension to Wires.

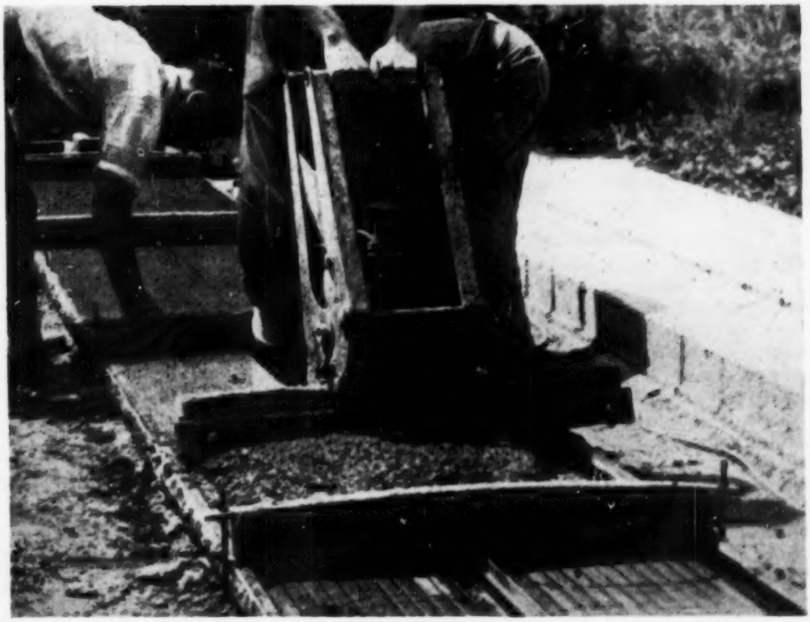


Fig. 6.—Stressing Bed for Slabs.

Hollow Insulation Beams.

Hollow blocks with thin walls, as shown in *Fig.* 3(*b*), are made on a machine which consolidates the concrete by vibration. Expanded clay, sand and cement are used in the proportions of 6 : 1 : 1. The blocks, when mature, are laid end to end to form the required length of beam, on a level bed between fixed anchor frames. The last few inches of the end blocks of each beam are filled solid. The prestressing wires are assembled as for the channel-beams, laid in the central groove of the line of blocks, and tensioned in the same manner. Concrete is then filled into the central groove and the flat upper part to form a tee-shape. Consolidation is effected by a vibrating hammer. Two beds, side by side and 135 ft. long, are in use for this type of beam. These beams are produced especially for use in dwellings and hospitals.

Prestressed Slabs.

Lightweight prestressed slabs, 1 ft. 6 in. wide by 1 in. thick increased to 2¼ in. thick at the edges and with rebated edges, are used for cavity walls and light roofs. The slabs are made on a double bed 135 ft. long (*Fig.* 6), using similar methods of assembling and tensioning the wires. The mould is of sheet metal on a framework mounted on springs fixed by rag-bolts to the concrete floor. Consolidation is done by a vibrator which travels along the top of the double mould and which screeds the surface as well as vibrating the concrete.

Concrete.

The concrete for the channel-beams and slabs is either a 1 : 1½ : 3 mixture with rapid-hardening Portland cement or a 1 : 2 : 4 mixture with high-alumina cement. In both cases gravel of ⅜ in. maximum size is used. With rapid-hardening Portland cement the wires can be released four days after casting and with high-alumina cement the wires may be released the day after casting. A crushing strength of 6-in. cubes of 6000 lb. per square inch is required before the wires are released, and the working stress in compression is 2400 lb. per square inch.

Grain Store.

A building in which the channels and the slabs have been used is shown in *Fig.* 7. This is a grain mill and store for Messrs. Hasler and Co., Ltd., of Dunmow for their Ingatestone branch, the architect being Mr. A. E. Wiseman, F.R.I.B.A., and the contractors Messrs. Tom Green (Building), Ltd. The building is about 140 ft. by 50 ft. and is three stories high, comprising a steel frame supporting floors of prestressed precast channel-beams designed to carry 3 cwt. per square foot on a span of 12 ft. 9 in. The external walls are of cavity construction formed by two leaves of prestressed precast slabs 1 in. thick laid in grooves in the in-situ concrete casing to the stanchions at 17 ft. 9 in. centres; between the slabs is a 1¼-in. cavity. The slabs are arranged so that the horizontal joints are staggered in the two leaves. The building contains two silos, of which the larger consists of fourteen bins in a double row. Each of the bins is about 8 ft. square by 26 ft. deep. The walls are 4 in. thick, comprising two leaves of slabs, the 2-in. space between the slabs being filled with concrete. The walls are supported at the bottom by steel beams from which are hung sheet-steel pyramidal-shaped bottoms. At the vertical junctions of the walls are in-situ concrete posts into which project the high-tensile wires from the ends of the slabs; additional reinforcement extending into the in-situ filling between the slabs, to tie the walls together at the junctions, is also provided in the posts.

Fig. 7.—A Grain Store with Precast Walls and Floors.

Lean Concrete Compacted by Roller.

In the year 1952 the Crawley Development Corporation constructed 5½ miles of estate roads. As the roads had to be completed by the autumn and hardcore was difficult to obtain, most of them were of lean concrete which was consolidated with a road roller. The carriageways are 22 ft., 16 ft., or 13 ft. wide and the concrete is 8 in. thick except for bus routes where it is 10 in. thick. The concrete was placed in two equal layers, each evenly spread with an extra ½ in. to allow for compaction. Care was taken to avoid segregation by additional mixing by hand where necessary. The concrete was compacted between steel side forms with a 2½-tons roller. Heavier rollers, up to 6 tons in weight, did not give better results than the lighter roller. *Fig.* 1 shows part of the road during construction.

Fig. 1.—Road During Construction.

The ratio of aggregate to cement was 14 to 1 by weight. The water-cement ratio was 0·7, including an allowance of 0·1 for losses due to evaporation during the placing of the concrete during the hot summer. The aggregate was graded ballast 1½ in. maximum size with about 35 per cent. passing a 3/16-in. sieve. Two continuous mixers were used, each with a capacity of about 180 cu. yd. per day.

No expansion joints were formed. Construction joints were formed by extending the bottom layer 2 ft. beyond the top layer, the upper layer being finished with a vertical face against a stop-end. After it had been rolled the surface was immediately sprayed with bituminous emulsion ; any small areas left unsprayed for convenience of rolling on the following day were covered with wet sand or waterproof paper. No traffic was allowed on the concrete until seven days after spraying, and for a further seven days traffic was restricted to that used for laying and rolling the tarmacadam surface. About 13,000 cu. yd. of concrete were placed in ten weeks.

The cost of this type of construction was about the same as that of a similar surface on hardcore. The ratio of the cost of labour, plant, and materials was 10 : 18 : 72. Measurements made by the Road Research Laboratory show that the

riding quality of the road is markedly superior to that of a similar surface on a hardcore base.

The work roads were built under the direction of the late A. J. W. McIntosh, then Chief Engineer of the Corporation. The contractors were Messrs. G. Wimpey & Co., Ltd.

The Resistance of Concrete to Impact.

In the U.S.A., the National Bureau of Standards, in co-operation with the Navy Bureau of Yards and Docks, is studying the properties of concrete under impact. In the results so far obtained, reported in the Bulletin for April, 1953, of the American Society for Testing Materials, the dynamic compressive strength of concrete was found to be up to 84 per cent. higher than the static strength, and the modulus of elasticity up to 47 per cent. greater.

In the investigation 3-in. by 6-in. cylinders were subjected to standard static tests in an hydraulic machine with a capacity of 60,000 lb., and to impact tests in a drop-hammer machine at various rates of loading up to 10 in. per second. The total duration of the impact corresponding to the highest rate of loading was 0·0003 second. Impact tests having a duration of about 1 second were made in an hydraulic machine operated at full speed. Two types of concrete were used with nominal static compressive strengths at 28 days of 2500 lb. and 6500 lb. per square inch. The dynamic compressive strength of a given type concrete was found to be higher than the static compressive strength, becoming relatively greater as the duration of impact decreased. The ratio of the dynamic to static strength for the weaker concrete ranged from 1·09 for an impact of 0·9 second duration to 1·84 for an impact of 0·00025 second duration. The corresponding ratio for the stronger concretes ranged from 1·13 for the longer impact to 1·85 for an impact of 0·00043 second duration.

The ratio of dynamic to static modulus increased as the duration of impact decreased, reaching a maximum of 1·47 for the weaker concrete and 1·33 for the stronger concrete. The linear portions of the stress-strain curves became longer and steeper as the duration of impact was decreased. The ratio of strain energy absorbed under dynamic loading to that absorbed under static loading reached a maximum of about 2·2 for both types of concrete tested.

Lectures on Building.

The following lectures have been arranged by the Ministry of Works. Admission is free.

Prestressed Concrete, by J. S. Arlett. Police Assembly Room, Shakespeare Street, Nottingham. October 13. 7.15 p.m.

Structural Use of Reinforced Concrete in Building, by J. G. Veryard. Hope Church Hall, High Street, Merthyr Tydfil. October 15. 7 p.m.

Soil Mechanics in the Building Industry, by A. L. Little. College of Arts and Technology, The Newarke, Leicester. October 21. 7.15 p.m.

Floor Finishes, by W. J. Warlow. Dudley and Staffordshire Technical College, The Broadway, Dudley. October 22. 7.15 p.m.

Concrete Placing and Formwork, by A. B. Harman. Walsall Technical College, Wisemore, Walsall. October 27. 7.15 p.m. Also by S. White. Howgill Street School, Whitehaven. October 28. 7 p.m.

Soil Mechanics in the Building Industry, by I. K. Nixon. Technical College, Mayfield Hall, Pelham Road, Gravesend. October 27. 7 p.m.

Dampness in Buildings, by A. G. Day. Courtroom, Guildhall, Wrexham. October 28. 7 p.m. Also at Old Church Hall, Rhiw Road, Colwyn Bay. October 29. 7 p.m.

MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word : minimum, 7s. 6d. Situations Vacant, 4d. a word : minimum 10s. Other miscellaneous advertisements, 4d. a word : 10s. minimum. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATIONS VACANT. Reinforced concrete engineers require designer-draughtsmen, preferably A.M.I.Struct.E., for work at Bristol or near London. Opportunity to widen experience. Good salary to right men. Box 3648, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Old-established Tees-side firm requires section leader reinforced concrete designer-draughtsman, fully experienced in designing and detailing reinforced concrete structures, foundations, and other civil work. Apply, giving full particulars and experience, quoting reference D, to Box No. 3656, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Consulting structural engineer, Westminster, requires experienced reinforced concrete draughtsman-detailer. High salary and good prospects for suitable applicant. Write, stating age, qualifications, and full details of experience. Box 3679, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. PETER LIND & CO., LTD., Civil Engineering Contractors and Reinforced Concrete Specialists, of Romney House, Tufton Street, Westminster, S.W.1, require qualified designers in their drawing office. Candidates, who should have at least three years' experience, to apply in writing, stating age, experience, and salary required.

SITUATIONS VACANT. AIR MINISTRY WORKS DEPARTMENT requires in London structural engineer designer/draughtsmen experienced in reinforced concrete or structural steelwork. Applicants should have had a sound technical training and several years' varied experience in design and detailing : (*a*) Reinforced concrete construction for all types of buildings, or (*b*) Steel-framed sheds, warehouses, and kindred types of buildings. Salaries £623 to £733 p.a.; starting pay dependent upon age, qualifications, and experience. Extra duty allowance payable. Reasonable prospects of promotion. Posts temporary but many have long-term possibilities, and competitions are held periodically to fill established vacancies. Apply, quoting CA and stating age, qualifications and previous appointments, giving dates and stating types of work done, to AIR MINISTRY, S.2(h)/NA, Cornwall House, Stamford Street, London, S.E.1.

SITUATION VACANT. Reinforced concrete engineers require section leader with first-class experience in the design of all types of reinforced concrete structures. The vacancy is in London. Five-days' week and pension scheme. Apply in writing, giving full particulars of age, experience, and qualifications, to Box C.E. 630, 191 Gresham House, London, E.C.2.

SITUATIONS VACANT. THE TRUSSED CONCRETE STEEL CO., LTD., Truscon House, 35-41 Lower Marsh, London, S.E.1, have vacancies in the London Office for a senior designer, two designers, and three designer-detailers with considerable experience in reinforced concrete D.O. work. Five-days' week and pension scheme. Apply in writing to the above address, giving full particulars of age, education, and previous employment.

SITUATION VACANT. Experienced reinforced concrete designer required for work in the project section of a company of reinforced concrete engineers and contractors. Five-days' week and pension scheme. Write, giving full details of age, experience, and qualifications, to Box C.E. 578, c/o 191 Gresham House, London, E.C.2.

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SITUATION VACANT. Draughtsman with at least three years' experience in reinforced concrete detailing required immediately. Apply in writing only to PETER LIND & CO., LTD., Romney House, Tufton Street, Westminster, S.W.1, stating age, experience, and salary required.

SITUATIONS VACANT. MINISTRY OF WORKS, STRUCTURAL ENGINEERING DIVISION. DRAUGHTSMEN required at Edinburgh. Maximum salary £612 per annum; starting pay according to age, qualifications, and experience. Experience in design and detailing of reinforced concrete or structural steelwork essential. Apply in writing, giving age and full details of experience, to THE SECRETARY, MINISTRY OF WORKS, 122 George Street, Edinburgh, 2.

SITUATIONS VACANT. MINISTRY OF WORKS. Assistant civil engineers are required for work at Windscale, Cumberland. Candidates must be of British birth, must have experience of work on site, and have passed : (*a*) Sections A and B of the examination for Associate Membership of the Institution of Civil Engineers ; or (*b*) Examinations recognised by that Institution as granting practical exemption from Sections A and B. Although these are not established posts, some have long-term possibilities. Hostel accommodation is available. Salary range £645–£960 per annum according to age and experience. Possibility of promotion to Main Grade ; salary range £960–£1,240 per annum. Stage age, nationality, details of training, experience, to W.G.10/C.W.E., MINISTRY OF WORKS, Abell House, John Islip Street, London, S.W.1.

SITUATIONS VACANT. Applications are invited for reinforced concrete design engineers, concrete control engineers, and civil engineering draughtsmen. Write, giving details of age, education, experience, and salary required to R. M. DOUGLAS (CONTRACTORS), LTD., 395 George Road, Birmingham, 23.

SITUATION VACANT. Consulting structural engineer, Westminster, requires senior designer-draughtsman with first-class experience in reinforced concrete for responsible position. Experience in structural steelwork an advantage. High salary and good prospects for suitable applicant. Write in confidence, stating age, qualifications, and full details of experience. Box 3685, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

(*Continued on page liv.*)

MISCELLANEOUS ADVERTISEMENTS.

(*Continued from page liii.*)

SITUATIONS VACANT. Applications are invited for several posts as assistant research and development engineers to work on various aspects of concrete materials and methods of construction in the Research and Development Division of the Cement and Concrete Association near Slough. Candidates should preferably possess an Honours Degree in Civil Engineering and be no more than 27 years of age. Preference will be given to those who have had one or two years' practical experience. Salary according to qualifications and experience, but would normally be from £450 to £700. Good prospects and opportunities for obtaining specialized knowledge. Pension scheme. Applicants should write giving full particulars to THE SECRETARY, CEMENT AND CONCRETE ASSOCIATION, 52 Grosvenor Gardens, London, S.W.1.

SITUATIONS VACANT. J. C. GAMMON (England), LTD., need competent civil engineers for service in India, Pakistan, and Malaya. Senior and junior engineers required for contract work chiefly in concrete construction. Salaries from £1200 to £2400 offered with excellent contract leave conditions. Apply Burwood House, Caxton Street, London, S.W.1.

SITUATION VACANT. Assistant required in consulting engineer's office. Applicant must be experienced in reinforced concrete detailing. Knowledge of design an advantage. Apply to JAMES WARDROPPER, 116 Victoria Street, London, S.W.1.

PART TIME AND EVENING WORK. Experienced reinforced concrete detailer required capable of working from calculations without supervision in own home. Jobs by agreed lump sum. Apply stating experience and approximate hours available per week. Box 3686, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

PROFESSIONAL SERVICES.

PROFESSIONAL SERVICES. Reinforced concrete designer-draughtsman with 20 years' experience in the design of reinforced concrete structures, foundations for heavy industrial structures, chemical plant, etc., undertakes to prepare complete designs, calculations, and working drawings to help consulting engineers. Excellent references. Moderate fee. Box 3683, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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FOR SALE. Concrete vibrators (pneumatic, electric, and petrol driven) for hire. CONTRACTORS SERVICES LTD., 65 Effra Road, London, S.W.2. Telephone: BRIxton 4081.

FOR SALE. Steel plate to sizes, pressed sections, discs and general metalwork. Keen prices and delivery. E. STEPHENS & SON LTD., Bath Street, London, E.C.1.

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